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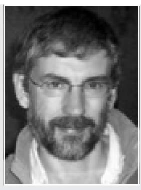
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Limit-equilibrium assessment of drystone retaining structures

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A limit-equilibrium analysis program has been developed as part of an investigation into the stability of drystone retaining structures. Initial verification of the program's function was in relation to field trials conducted in 1834 by Lieutenant-General Burgoyne, which have been the main reference to date for checking numerical modelling of drystone retaining walls. Parametric studies and investigations of bulging mechanisms are reported and analysed. Program predictions have been compared with the initial results from new small-scale and full-scale drystone retaining wall tests.

NOTATION

h	horizontal spread of surcharge per unit depth
k_a	coefficient of active pressure
$\boldsymbol{P}_a$	earth pressure
$\boldsymbol{P}_H$	horizontal component of $\boldsymbol{P}_a$
$\boldsymbol{P}_V$	vertical component of $\boldsymbol{P}_a$
v	depth of surcharge within backfill below point of application
W	self weight of wall block
α	inclination of internal wall face
δ	incident angle of $\boldsymbol{P}_a$ on internal wall face
ε	eccentricity of thrust line

1. INTRODUCTION

Drystone technology is an ancient form of construction, suitable for applications ranging from simple field walls to large earth-retaining structures several metres high. Typically, it utilises undressed stone and is constructed without mortar; structural integrity is maintained through self-weight, inter-block friction, and overlapping of stones. The technique relies upon the skill of the mason in selecting a suitable stone for each location in turn, placing each appropriately.

There are estimated to be some 9000 km of drystone retaining structures lining the road and rail networks[1] of the UK, while globally the total length is many times this figure, with walls found throughout Europe, parts of Asia, Africa and the Americas. Most construction in the UK dates to the nineteenth and twentieth centuries. Though poorly constructed walls presumably collapsed shortly after their construction, the majority of walls have remained perfectly stable over decades of usually steadily increasing loading and weathering of the constituent stone. However, many otherwise stable walls have deformed or bulged, and with little guidance currently available to assist engineers in the assessment of these structures,[2] the authorities responsible for any potentially unstable walls are often forced to replace or rebuild them—in many cases unnecessarily and at great cost. The total replacement cost for the walls lining the UK's highways is estimated to be in excess of £10 billion.[3]

Such figures highlight the need for the means to assess these structures adequately, as current design standards often deem them insufficiently safe.[4] There are several difficulties when attempting to assess drystone structures: often, very little is known about any particular wall, as their construction usually predates the strict design guides that are adhered to today, leaving uncertainties regarding wall thickness, age, construction quality, foundation capacity and the mechanical properties of the material retained. There are also regional differences, as many styles of wall construction exist, either necessitated by the material properties of the stone or for purely aesthetic purposes.

There is in any case an important philosophical difference between assessing an existing structure and designing a new one, in that many of the uncertainties that are to be covered by factors of safety in design have been resolved by the fact that the wall has been standing and has remained serviceable. While this fact gives no assurance that the structure has ever experienced a full design applied loading, there remains the important fact that the assessor is most concerned by possible changes from the status quo, in which the factor of safety must at least exceed 1 under permanent loading conditions. Inappropriate interventions such as pointing become of greatest concern, because while they may increase the compressive strength and stability at the face of the structure, they can lead to catastrophic changes in the pore water pressure regime. It is therefore very desirable to be able to assess the possible impact of changes in geometry and loading on the structural stability of an existing wall, especially given that old structures often appear to have departed from their originally constructed geometry.

It is also important to understand the extent to which structural stability is dependent upon precision in geometry and quality of construction. Standards for modern drystone retaining wall construction are very high, with good practice resulting in very strong structures with a high degree of integrity. However, such construction is time-consuming and expensive. A proper understanding of drystone retaining wall stability could lead to narrower structures requiring less volume of carefully placed stone and significantly less construction time. Similarly, an understanding of their tolerance of deformation and of the actual sensitivity to variability within the construction could lead to faster construction. These factors would make it easier to repair drystone structures rather than replace them, and easier to replace with a new drystone structure that will be sustainable, reusing materials where possible or using locally sourced materials, and resulting in structures that are in keeping with their surroundings. It would also make this highly sustainable form of construction a more attractive proposition for new constructions.

2. OBJECTIVES AND SCOPE

As part of an ongoing investigation, several unmortared retaining structures of both large and small scale have been built and tested to failure. These experiments are carefully monitored, and both the stress changes and deformations at critical locations are recorded, and then used to determine the underlying mechanisms behind the failures. In parallel with these studies, the authors are developing alternative means of analysing drystone structures, which are then verified with the gathered physical test data.

Current analysis techniques for drystone walls are either simplistic, by considering the static equilibrium of the wall as a monolithic structure, or too complicated, using time-consuming numerical packages to model each element within the wall and backfill. Numerical packages such as UDEC (universal distinct element code) may provide precise details regarding wall stability and the potential failure mechanisms, given sufficient data and careful modelling, but the analysis can take several hours, making parametric studies of any particular structure a lengthy and expensive process.

Neither option is acceptable for routine use. A computer program has therefore been developed to explore efficient approaches to analysis and design that might be carried out by hand calculation, or by a range of simple computing approaches. The program is based on a rapid two-dimensional limit equilibrium (LE) appraisal for structures of any size, with the ability to account for any deformations or bulges that might occur. In addition, this program is being utilised to further understand the mechanisms behind the deformations within drystone walls, as well as the critical factors that affect this particular construction.

3. DRYSTONE CONSTRUCTION

Although many differences exist between the various drystone construction styles, several common features are usually exhibited. Typical drystone walls are built in horizontal layers or courses, with each course ideally consisting of stones of a uniform thickness, retaining a straight and level appearance. The cross-section of the wall usually consists of a well-made, tightly packed outer face, with a core of smaller random material packed behind. Some drystone retaining walls follow this core material directly with the retained backfill material, whereas others have a second inner face, usually less well finished than the outer face. Tie-stones span from the outer to the inner face, binding the wall together. Where there is no inner face, tie-stones are often used to anchor the outer face further back into the packing fill. Coping stones can act in a similar manner, spanning the entire width of the wall at the crest (Figure 1).

Each block within the wall should ideally be in contact with several other stones, and pressure upon any part of a freshly placed stone should not cause any rocking or lifting at the opposite corner. In practice it is usually necessary to wedge in small pieces of rock, known as ‘pins’, to prevent rocking. The unavoidable presence of these pins presents a weakness for all drystone structures, especially as weathering of these smaller elements has a substantial effect much more quickly than for the larger stones. Pins are often used to allow a more even appearance to the face, and assist in drainage by tilting stones so that their outer surface is in the plane of the face. Thus the face of a structure can often give a misleading impression of a very tight, well-ordered construction, while behind the face there are substantial voids held open by a large number of small pins.

Depending on the quality of workmanship and the material used, the density of the walls can differ vastly. Void

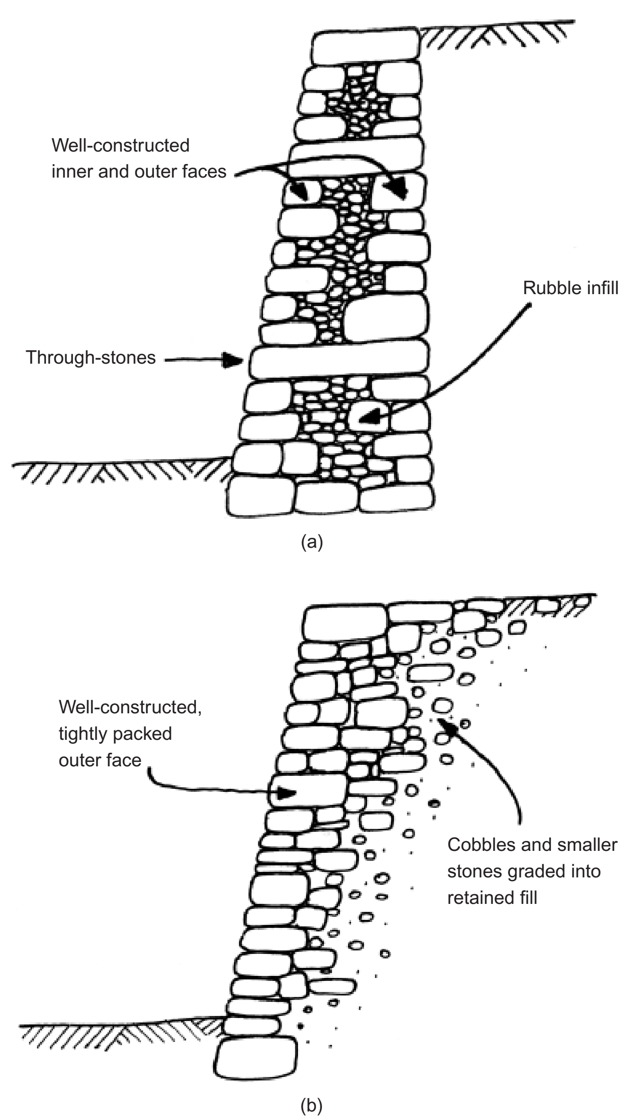


Figure 1. Comparison between (a) double-faced and (b) single-faced walls

percentages within drystone walls have been generally considered to range between 10% and 20%;[5] however, measurements carried out within this project on a range of test sections showed these values to be too low. Sections of wall were built to various standards by the project masons within timber boxes. As the density of the wall material is known, together with the overall volume of the sample, a stone/void ratio can be easily determined. A very carefully constructed, tightly packed, double-faced wall with almost ideal Cotswold limestone has around 20% voidage, while over 40% is possible within poorly built walls. The consequences of a high void ratio are more extensive than just reduced weight of the structure: a loosely packed wall gives the blocks within it a much greater opportunity to rotate and slide, facilitating bulging and other deformation, or even collapse.

4. PREVIOUS WORK

To date, despite its widespread use, only a limited number of investigations into drystone behaviour have been conducted. Until recently, the only physical test data for full-scale drystone walls dated back over 170 years to work conducted by Lieutenant-General Burgoyne in 1834.[6] Burgoyne built four full-scale granite walls, up to 6·1 m tall, 6·1 m long, and of varying thickness, in an attempt to quantify the effect that the wall profile has upon stability. These walls were then gradually backfilled until either full retention was achieved or collapse occurred. Movements and general observations were recorded upon the placement of each layer of fill, but only reported posthumously in 1853 from Burgoyne's notes.

Based on these field tests, several numerical studies have recently been conducted. UDEC has been used by various authors[1,5,7–10] both to test the validity of various modes of analysis and to study further the various parameters at work within drystone structures. Although highly informative, these investigations are both complex and time-consuming, often requiring several hours to run a single cycle of analysis. Work is currently being carried out in conjunction with this project to develop three-dimensional models of the full-scale tests described in this paper.

5. PROGRAM OPERATION

By analysing the stabilising forces within the wall, and using Coulomb's earth pressure coefficients to determine the horizontal and vertical stresses acting at each level up the back of the wall,[11] the magnitude and direction of the overall thrusts are determined (Figure 2). The initial wall geometry is entered, along with the material properties of both the wall and the backfill (mass, friction angles, etc.), and the eccentricity (ε) is calculated at a number of levels to generate a thrust line, as shown in Figure 3.

In addition to forces arising from the self-weight of the backfill, patch surcharging may also be applied. Additional pressure is then applied to the backfill, spreading out by a ratio of 1H : 2V. This is clearly a simplification, as used for example in BS 8006,[12] compared with the more rigorous approach of Bolton[13] as suggested in BS 8002,[4] but for the present purposes this approximation allows the combination of rapid calculation and reasonable accuracy required here. Further justification of this approach was given by Corte.[14] It is currently assumed that the surcharge will have no effect upon the calculated

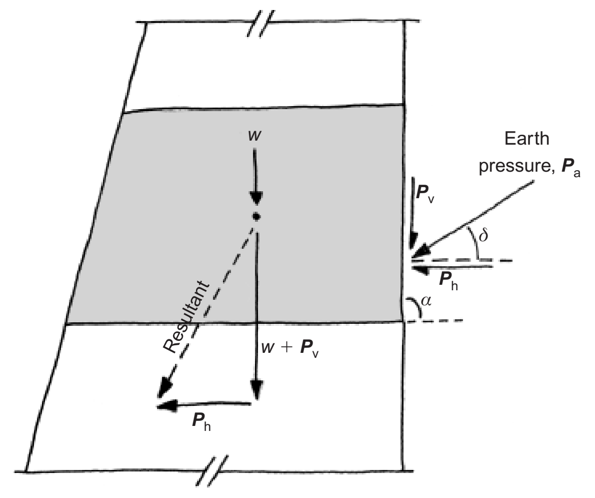


Figure 2. Determination of resultant forces

thrust line until the expanding area over which it is distributed crosses the boundary of the wall. Although the analysis is two-dimensional, three-dimensional load dissipation can be assessed as spreading in both horizontal dimensions. The most problematic loading is wheel loading from a heavy vehicle, so it is important to model the three-dimensional distribution, even if crudely. A more sophisticated stress distribution calculation is simple to implement, but given the uncertainties in wall and backfill stiffness and anisotropy, this may overcomplicate the analysis without adding value.

Standard masonry construction recommends that for stable construction the eccentricity of the thrust line must remain within the middle third of the structure ($\varepsilon = \frac{1}{6}$ of the base width away from the neutral axis). If the masonry were to behave in a linear elastic manner, and deformations were very small, this would result in no tension being taken at the back of the structure. However, even if the tensile strength of the masonry were zero, a thrust line in front of the middle third would simply result in the stone at the back of the structure being progressively unloaded, which need not have any immediate serious consequences. As the thrust line moves further forwards, the area carrying the vertical load reduces, so increasing the stress. The compressive strength of most masonry, including drystone, is usually relatively high compared with the stresses acting. Therefore compression failure of the main stones is very unlikely, but a concentrated thrust may cause localised crushing of weakened pins or a flexural fracture of some stones, leading to further deformation. In addition, foundation settlement might give rise to significant deformations. Given sufficiently strong masonry and foundation, failure would occur only once this thrust line breached the wall face ($\varepsilon > \frac{1}{2}$). However, individual block rotation will occur before this,[5] as the block at the face must carry all of the lateral thrust, as well as a vertical load that is moving closer and closer to its leading edge. This could result in an immediate rotational failure of the entire structure, and indeed such a mechanism accounts for the heights reached by the Burgoyne walls that failed.[5] It may also be noted that a crushing failure at a point of contact or of a pin may lead to collapse before the line of thrust reaches the front face.

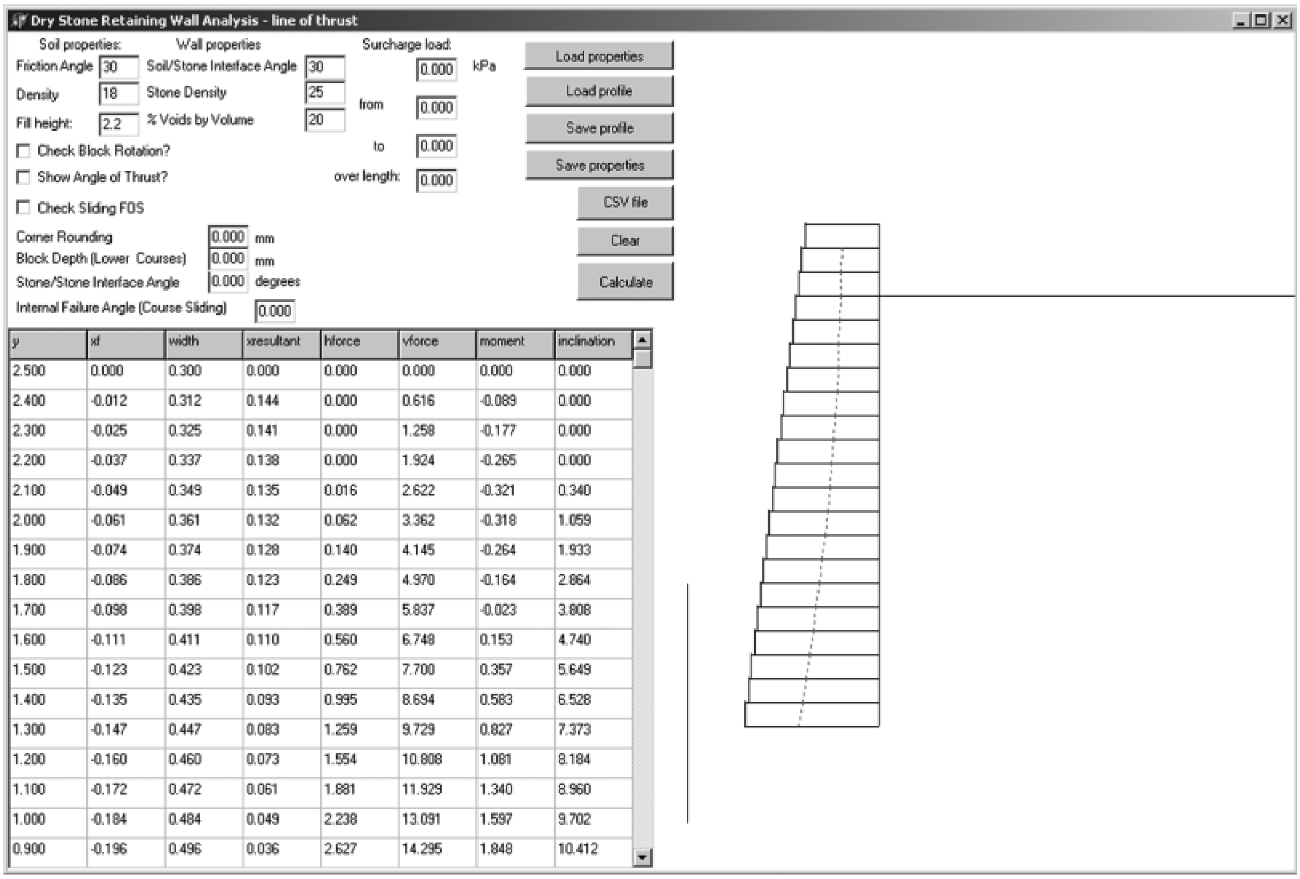


Figure 3. Program operation screen

A significant advantage of the program is the ability to allow the user to deform the wall model and induce bulges. Once the parameters are set and the wall profile is generated, the user may move the individual blocks to any position, either by typing in a new coordinate, or by clicking the cursor at a new position for the front face. Recalculation and redisplay of the new thrust line is virtually instantaneous (Figure 4). In such a manner, idealised wall structures can be deformed to represent commonly observed drystone bulging patterns, to assess their effect upon overall stability. Conversely, existing walls that have bulged may be quickly recreated using the program to ascertain their stability.

6. PROGRAM VALIDITY CHECK

Initially, the program was validated against Burgoyne's four test walls. The geometries of each wall were recreated, and the material properties entered from Burgoyne's tests.[6] Backfill heights were then systematically increased by 300 mm (simulating Burgoyne's test procedure) until the thrust line reached the wall face, indicating failure via toppling. The final simulated heights were very similar to those recorded by Burgoyne, and indeed also similar to previous attempts using other more sophisticated and complex numerical packages[8] (Table 1).

Both the first and second of Burgoyne's test walls were backfilled to their full height without excessive movement, and by using the LE program it can be demonstrated that the thrust line lies within the boundaries of the wall. For both these walls the eccentricity is outside the middle third at the base, indicating uplift at the heel. The third and fourth walls both fell before full height of retention was achieved. For both these wall geometries the LE program predicted failure at a height similar to that found by Burgoyne, although it has been demonstrated that consideration of individual block rotation gave a tighter correlation with actual failure heights.[5] To allow this to be seen in the program, the direction of the resultant force at each level is also shown at the point at which it acts. A resultant that points in front of the toe of an individual block may result in rotation of that block. With regard to the results shown in Table 1, this would indicate a failure at 5·2 m for wall C, bringing it in line with the observed results.

7. PARAMETRIC ANALYSIS

Thanks to the nature of the program, a parametric analysis of any structure is a rapid process. This has a twofold application: first, it allows users to quickly grasp which parameters have the greatest impact upon wall stability; and second, it allows engineers in the field a greater flexibility when assessing existing walls. Once the cross-section of a wall has been recreated within the program, each variable may be altered to examine the safety factors at the worst possible conditions.

From the program it is apparent that, for any given geometry of wall, several parameters are dominant for stability. For example, the assumption of a 1H : 2V load spread from surcharges means that the loads must be close to a wall to have an effect, but it is also found that loads must be relatively large, corresponding to wheel loads from the heaviest trucks. This corresponds with anecdotal evidence, confirmed by

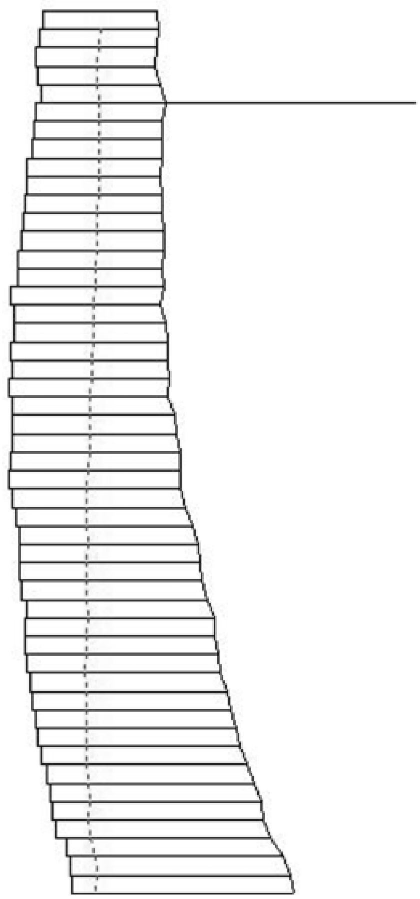


Figure 4. Example of user-defined bulging

numerical modelling studies,[9] that there is a relationship between increasingly heavy traffic and failures of walls that had been safe for many years.

The friction angle of the backfill material is critical to wall stability. This angle determines the coefficient of active pressure (k_a), which in turn determines the magnitude of the horizontal forces upon the retaining wall. A stiff, tightly packed backfill material might have a high density, but its consequently high angle of friction is likely to result in a lower horizontal pressure than a much looser, yet less dense, material.

There is also the friction between the wall and backfill to consider. Because of the nature of drystone construction, the wall faces are generally rough, which allows the inner wall face adjacent to the retained material to attract some of the vertical load from the backfill. As this vertical force acts against the overturning forces and stabilises the wall, this is a value that would ideally be as high as the interface allows, although in reality it is not always guaranteed that the full friction angle will be achieved.

One of the most variable and difficult to ascertain parameters is the density of the walls themselves. Non-destructive testing methods, such as ground-penetrating radar, or horizontal coring can be used to give some indication of wall depth, profile and even voidage. While the density of the rock will not vary greatly, its age, the construction style, and the skill of the mason will all affect a wall's overall density and hence the total volume of voids within. While this voidage has little impact on wall stability when changed by a few per cent, the value may vary by much more than this, as mentioned above. Low density reduces the wall's stability in terms of both sliding and overturning. Perhaps most critically, a reduction in density allows easier movement and rotation of the individual blocks, determining the flexibility of the wall and the amount of bulging that may occur.

8. BULGING INVESTIGATION

Bulging is common in drystone walls, usually occurring at roughly a third to half the height of the wall, creating a distinctive 'belly bulge' shape (Figure 5). Upon investigation of the effects of bulging and wall deformation, it was discovered that, far from causing instability, a moderate bulge may indeed increase the safety of a wall against certain failure modes when subject to surcharge loading conditions.

Bulging begins when the loads behind the wall cause blocks or entire sections of wall to move, and the resulting movement causes both the forces acting on the wall and the equilibrium of its own mass to change, such that a new equilibrium position is found. Were this not the case, the wall would continue to move, resulting in collapse. This rearrangement usually occurs lower down the wall, and can be due to slips in the retained earth, increased pore water pressure or an increase in loading conditions, or the equilibration of negative pore pressures within the backfill. Bulging probably occurs much more commonly than is appreciated, but is usually on a scale too small to be noticed. Bulging and movement can also occur much higher up a wall—usually caused by localised surcharging, or disturbances to the wall itself, such as growth of vegetation, although this is generally detrimental to wall stability and can easily lead to partial or full wall collapse.

Once a bulge is formed, the pressures acting upon the wall must change in response to the new geometry. A section of a

Wall geometry	In situ observations	UDEC analysis	Limit equilibrium analysis	
	Maximum fill height: m	Maximum fill height: m	Maximum fill height: m	Eccentricity at base of wall: mm
Wall A	Full height	Full height	Full height	102 from toe
Wall B	Full height	Full height	Full height	156 from toe
Wall C	5·2	5·2	5·5	N/A
Wall D	5·2	5·2	5·2	N/A

Table 1. Comparison of numerical and limit equilibrium analyses with observed test data

Figure 5. Deformed wall

typically bulged wall is shown in Figure 6, highlighting the common features. Above the bulge, the wall is leaning back somewhat, having a twofold effect. First, it stabilises the wall by moving its centre of gravity away from the toe of the wall, which is usually the overturning point. Second, it reduces the magnitude of the forces applied to the wall by the backfill.

Below the bulge, the wall is leaning forwards, causing the active pressures within the backfill to have a much greater effect upon this portion of the wall. The magnitude of the force will be greater, but the downward component will be most increased, so increasing the stability of this portion of the wall, provided that the face has not moved so far forwards that individual blocks are no longer supported. Overall, these changes tend to be in favour of increasing wall stability, and walls have commonly remained perfectly safe for years while displaying this type of bulge without any detrimental effects. However, new works, such as excavations for services at the toe of deformed walls, or changes in loading, are common factors attributed to triggering collapse.

Because of the flexible nature of these walls, significant movements may take place before a failure occurs, giving visible warning signs. Final collapse can occur either by toppling or by bursting, but is usually a combination of both.

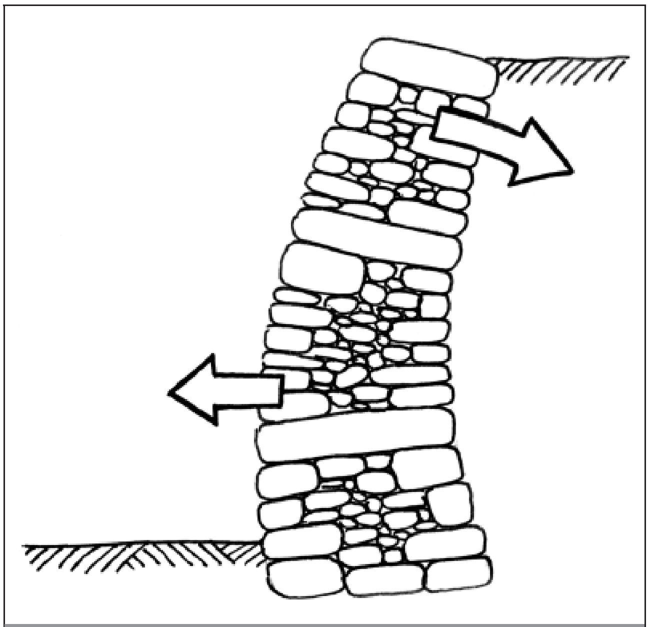


Figure 6. Section of typically bulged wall

9. SMALL-SCALE TESTING

A series of small-scale tests have been conducted to determine whether observed drystone behaviour can be recreated in smaller, simpler experiments. To house the tests, a steel box was constructed, with the capacity to hold scale walls 500 mm high and 500 mm wide (Figure 7). The box was lined with a double layer of plastic sheeting, to help reduce friction at the edges and hence minimise end effects.[15]

As the goal of these experiments was to reproduce full-scale drystone behaviour, small pellets (2–3 mm diameter) of lead shot were used as backfill to induce sufficient pressures to cause deformations and failures. The lead shot used has an uncompacted unit weight of 50 kN/m^3 and an internal friction angle of 31°, allowing the generation of sufficient lateral pressures to overcome the stabilising forces within the test walls.

Figure 7. Test set-up

To overcome three-dimensional effects long blocks were used, each spanning almost the whole width of the steel box but with gaps at either end to allow small rotations of the wall elements. Both timber and concrete block walls were tested independently. The timber blocks were quickly discarded as their densities proved too low for realistic modelling of drystone behaviour (5·5 kN/m^3 as opposed to 24 kN/m^3 for the concrete blocks), although the data proved useful for comparison with the LE program results.

For each test, the scale walls were fully constructed without any retained backfill, and then slowly backfilled. Results from the small-scale tests are shown in Table 2 together with the backfill heights predicted by the LE program.

From Table 2 it is clear that the program is accurately predicting the collapse heights of these small-scale tests. It was assumed that the interface friction between the wall blocks and the backfill was two-thirds the full value of the backfill's internal friction angle. Evidence gathered by the small-scale tests supports this assumption, although in practice it is difficult to ascertain precisely how much of the backfill's full friction angle has been mobilised against the wall. This obviously has a large impact upon wall stability, although it is expected that ground settlement over time and the rough nature of drystone structures result in the full friction angle being mobilised for in situ walls.

10. LARGE-SCALE TESTING

In addition to small-scale testing, full-scale walls have been built and tested to failure to validate modern theories and analysis tools. To test the drystone walls, a bespoke test facility has been constructed, allowing the re-creation of localised or general foundation settlement, backfill settlement and localised surcharging (Figure 8).

Each wall was constructed of Cotswold limestone by skilled masons. At 2·5 m high and over 12 m long, the test walls are large enough to be representative of many of the walls found throughout the country, and are built using traditional methods, including regular placement of through-stones and a line of coping stones at the peak of each wall. The first wall varied in thickness from 600 mm at its base to 400 mm at its peak, and was constructed to a high standard. The second wall was of an intentionally poorer quality, and 100 mm more slender throughout.

A large range of instrumentation is used to monitor each wall, including extensive surveying, multiple transducers, load and pressure cells, high-resolution imagery and video footage. With this vast range of data, the critical events that lead to the failure of each wall can be better understood and incorporated into both our general understanding and the theories and programs that are used to evaluate a wall's stability.

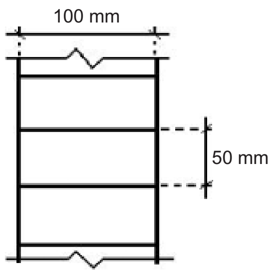
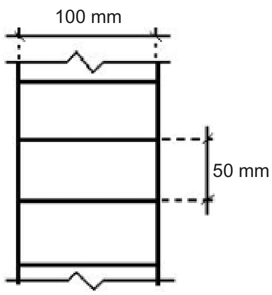
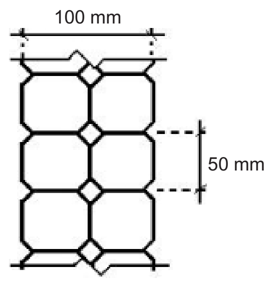
Cross-section profile	Wall material	Failure details
	Softwood timber Wall height: 500 mm Density: 5·5 kN/m^3 Friction angle: 24°	Recorded backfill height at failure via toppling: 245 mm Predicted backfill height at failure: 240 mm
	Concrete blocks Wall height: 500 mm Density: 24 kN/m^3 Friction angle: 29°	Recorded backfill height at failure via toppling: 350 mm Predicted backfill height at failure: 350 mm
	Concrete blocks (10 mm chamfer) Wall height: 500 mm Density: 24 kN/m^3 Friction angle: 29°	Recorded backfill height at failure via toppling: 300 mm Predicted backfill height at failure: 315 mm

Table 2. Small scale test details

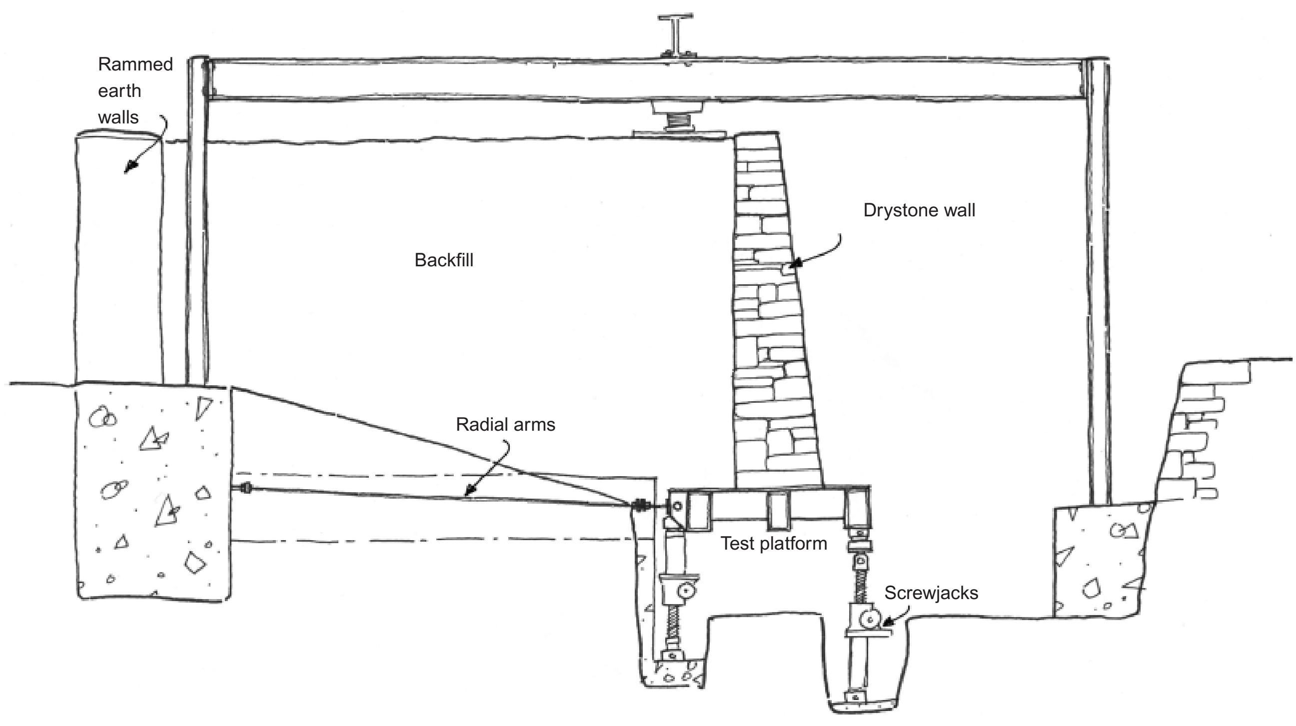


Figure 8. Bespoke test facility

For each test the LE program was used to check initial stability. Ideally, each wall should be able to retain the full height of fill intended with a sufficient margin of safety during installation of the remaining instrumentation and loading plates, while being close enough to its ultimate conditions that the proposed movements and loadings can take the structure to failure. Both walls complied with these criteria; however, the second wall was significantly less stable, with the eccentricity on completion lying in front of the wall's middle third.

The initial phase of each test to date has involved the raising up of the platform to ensure that the maximum possible friction is generated at the wall–backfill interface. In real walls, full friction is likely to be achieved, owing to settlement of the backfill following construction of the wall. In order to take full control over this important parameter the inverse movement is applied, and the wall is moved upwards in relation to the backfill via the jacked platform. Load cells supporting the wall show a steady increase as the platform is lifted, until the full friction is mobilised when the loads level out (Figure 9).

Following the initial raise of the platform, wall 1 was subjected to a combination of forward rotation and surcharging, while wall 2 was simply surcharged until failure occurred. Both walls failed by toppling, though each displayed a great deal of movement prior to collapse, including block rotation and sliding (Figures 10a, 10b and 10c).

Throughout each experiment the geometry of the wall face is constantly monitored, allowing the wall profile to be recreated within the LE program, so that stability can be assessed as the loads are changed and the walls deform. The images of the thrust lines generated immediately prior to failure for the first two tests are shown in Figure 11.

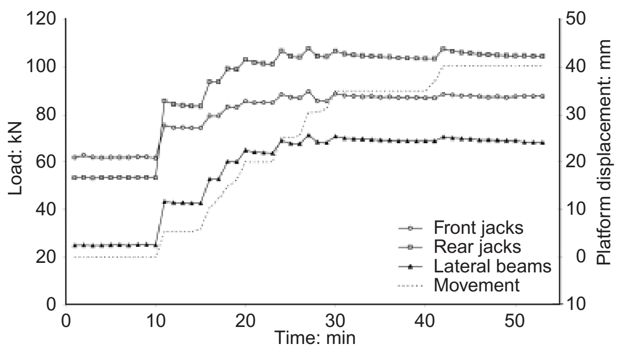


Figure 9. Load cell readings during platform raise

From Figure 11 it is evident that the structures are both on the verge of collapse, although three-dimensional effects may have given added stability, especially in the case of the first test wall. Both walls developed bulges only through the central region of the wall adjacent to where the surcharge loading was applied. The high friction generated between the courses allows a tensile strength to develop along the length of the wall, so the relatively lightly loaded end sections help support the central section. This load shedding subsequently allowed the first wall to deform to a greater extent than would have been possible had the wall been acting purely in plain strain, with the wall profile showing the coping overhanging the toe by some 500 mm prior to failure.

Following this experience, the second wall was intentionally of a poorer quality than the first, showing a number of running joints, a term used to describe the situation when the joint between two adjacent blocks is similar in position to a similar

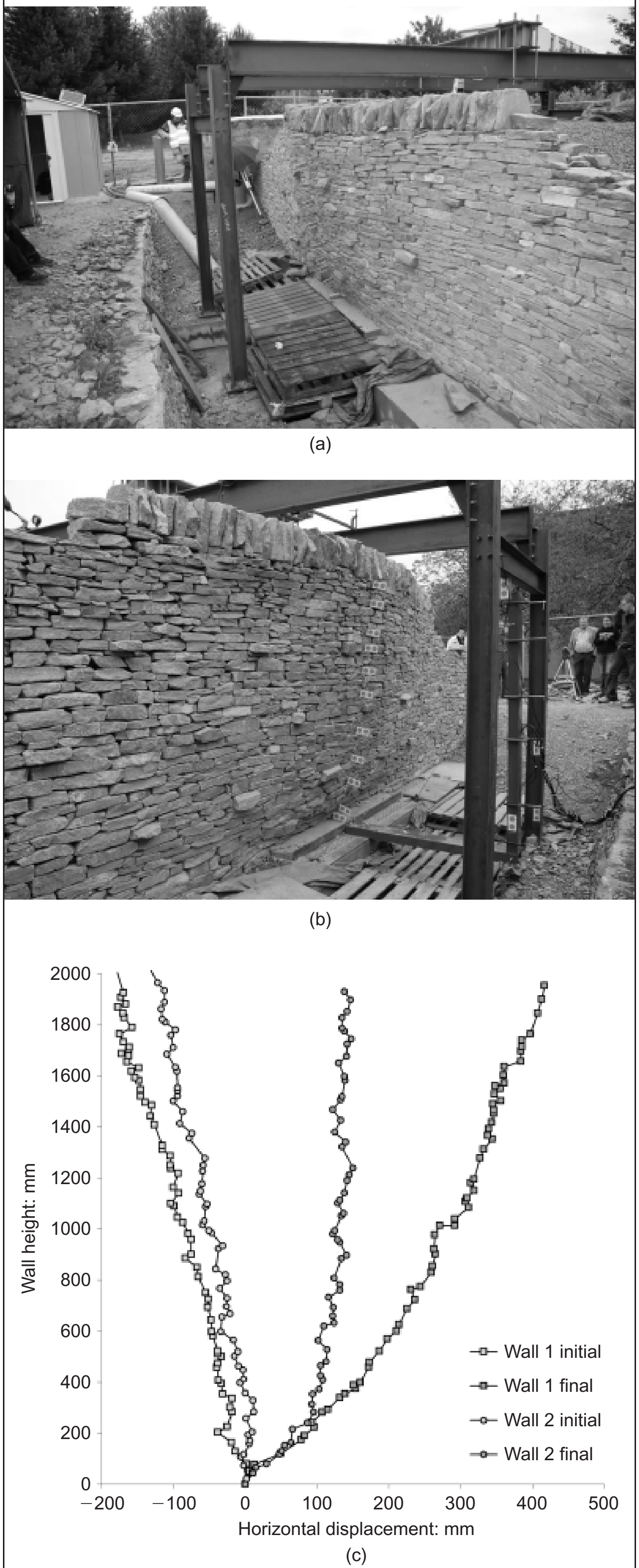


Figure 10. Physical test results: (a) wall 1 prior to failure; (b) wall 2 prior to failure; (c) graph of wall displacements

joint above or below. These prevented the transfer of load along the length of the wall, and opened up as loading proceeded, to allow the central section to move more easily relative to the adjacent sections. The result of this was that the wall behaved in a manner more akin to that represented in the LE program, and probably more akin to sections of wall that actually fail in practice. Most wall failures are localised at weak points, rather than being general failures along the full length of a wall. Thus three-dimensional effects, particularly the transfer of load along the line of a wall, can certainly help support a weak section, but failure is much more likely to occur where loss of such support results in a behaviour that is nearer two-dimensional or plane-strain, and two-dimensional analysis is therefore recommended for most situations. The final observations were recorded just minutes before failure, when the wall was remaining stable in the unloaded state but giving indications of imminent collapse (increased movement for only minimal surcharge loading), and this was successfully reproduced by the LE program. The surcharge loading was controlled by displacement rather than by load, allowing a progressive deformation past peak load with full control, and consequently safe collection of data, until eventual collapse associated with excessive distortion of the geometry of the structure. Both structures remained absolutely stable, with no ongoing deformation, when the surcharge loads were removed, even though they were very severely distorted.

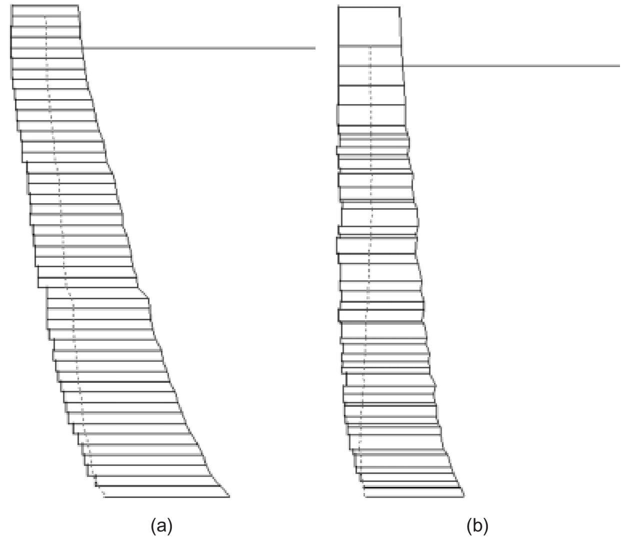


Figure 11. Limit equilibrium analysis of physical tests: (a) wall 1; (b) wall 2

11. CONCLUSIONS

The limit equilibrium analysis program described in this paper has enormous potential compared with numerical analysis packages. Its simplicity allows any engineer with a basic knowledge of a wall's geometry and material properties to obtain a reliable understanding of the factors influencing its stability, without the need for the detailed knowledge, advanced design parameters, time and expertise that are needed for reliable numerical analyses. The program's flexibility in use allows walls of any geometry with variable backfills to be analysed, and the application of surcharging can be applied to represent circumstances such as new constructions in the proximity of the wall or increased vehicle loading.

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